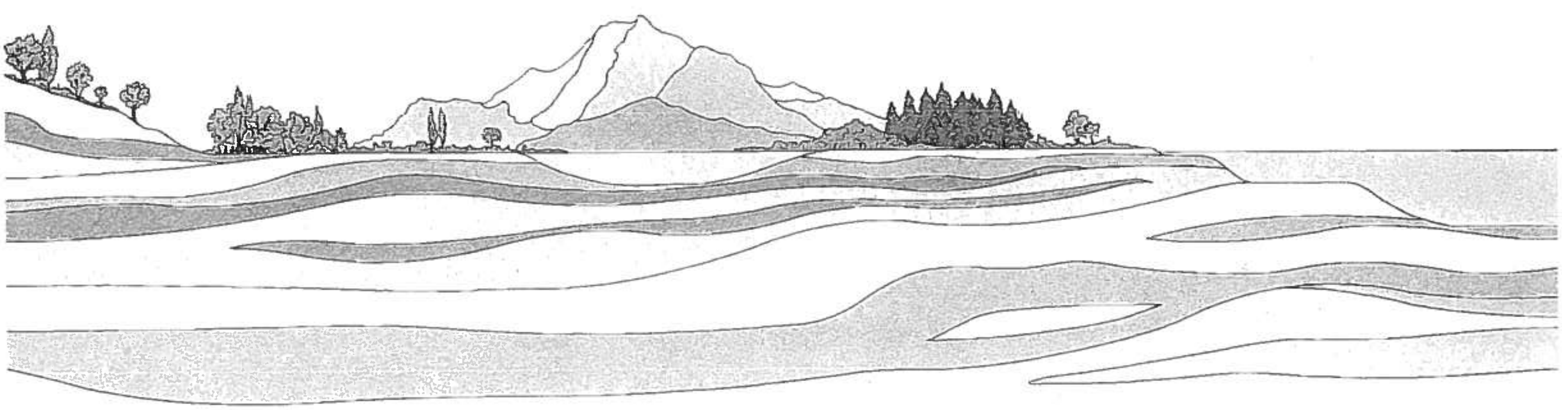
PRELIMINARY FOUNDATION REPORT CHAPALA STREET BRIDGE SEISMIC RETROFIT/REPLACEMENT PROJECT SANTA BARBARA, CALIFORNIA

Prepared for:
Drake Haglan & Associates

DRAFT

March 18, 2010



APPENDIX F

DRAFT GEOTECHNICAL REPORT

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March 18, 2010
Project No. 3713.001

Drake Haglan & Associates
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Attention: Mr. Craig Drake

Subject: Preliminary Foundation Report, Chapala Street Bridge Seismic Retrofit/Replacement Project, Santa Barbara, California

Dear Mr. Drake:

Fugro is pleased to submit this Preliminary Foundation Report for the seismic retrofit/replacement of the Chapala Street bridge across Mission Creek in Santa Barbara, California. This report was prepared according to our proposal dated November 19, 2009. Authorization for our services was provided through the Professional Service Agreement between Drake Haglan & Associates and Fugro dated February 8, 2010.

This report describes the general geologic setting and geotechnical conditions at the site primarily based on review of existing available data in the project area. The report also provides preliminary information and input regarding liquefaction potential and seismic settlement and potential foundation types for support of the bridge retrofit/replacement.

We appreciate the opportunity to work with Drake Haglan & Associates and the project team on the proposed Chapala Street Bridge Retrofit/Replacement Project in Santa Barbara, California. If you have any questions regarding the information provided in this report, please do not hesitate to contact me.

Sincerely,

FUGRO WEST, INC.

DRAFT

Gregory S. Denlinger, G.E. 2249
Principal Geotechnical Engineer

Copies: 4 – Addressee

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PLATES

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1. PROJECT DESCRIPTION

The project generally consists of retrofitting or replacing the existing pony-truss bridge spanning Mission Creek at Chapala Street. Based on discussions with the project team during the initial development process, we anticipate the project will consist of replacing rather than retrofitting the existing bridge. The replacement bridge will be a new concrete single span bridge constructed in the general area of the existing bridge. The bridge is located on Chapala Street immediately south of the Union Pacific Railroad (UPRR) corridor and west of Yanonali Street in Santa Barbara, California. The location of the site is shown on Plate 1 - Vicinity Map. The general layout of the site and proposed bridge is shown on Plate 2 - Field Exploration Plan.

Based on the Request for Qualifications (RFQ) dated August 28, 2009 prepared by the City of Santa Barbara, we understand the proposed bridge replacement is part of the City's long range plan to reduce the flood hazard on Mission Creek through the City of Santa Barbara and will be designed to accommodate other planned improvements related to the US Army Corps Lower Mission Creek Flood Control Project. The project will be incorporated into the proposed Phase 1A and Phase 1B channel widening projects planned upstream and downstream of the Mason Street Bridge and will need to consider the proposed Mission Creek by-pass culvert that will extend through the railroad depot and beneath the existing UPRR corridor. The by-pass culvert is currently in the design stage and the County of Santa Barbara Flood Control Division is the lead agency.

1.1 EXISTING FACILITY

The existing bridge trends through the intersection of Chapala Street and Yanonali Street and is at a highly skewed angle relative to the adjacent streets. Chapala and Yanonali Streets are two lane roads that primarily serve local residential traffic. Chapala Street dead-ends at the UPRR corridor on the northwest and Yanonali Street jogs to the northeast on the eastern side of the bridge.

According to the Caltrans Bridge Inspection report (Caltrans 2007) for the bridge, the Chapala Street Bridge was constructed in 1920. The bridge consists of a single span steel truss bridge with abutment walls consisting of mortared sandstone blocks. The bridge is about 80 feet long and 60 feet wide. Observations at the site indicate that the creek bottom is lined with concrete beneath the existing bridge. The bridge inspection report suggests the existing bridge is supported on shallow foundations and indicates that the footing for abutment 2 is exposed due to scour and removal of the concrete lining at that location.

As part of our work for the project, we advanced probes into the ground adjacent to the abutment wall in an effort to assess the lateral extent of the footing behind the abutment wall. Three probes were advanced to depths of 25 feet or greater on the east side of the bridge at distances ranging from about 5 feet to 21 inches from the abutment wall (estimated by the prominent crack in the pavement surface). The probes did not encounter any significant resistance indicating that the abutment footing at the location explored does not extend beyond

21 inches from the back of the wall. It should be noted that the conditions encountered at this location is representative of the foundation conditions in other areas of the abutment wall.

The terrain in the project vicinity is generally flat and the surrounding area consists of residential housing south of the Chapala and Yanonali Street intersection and the UPRR corridor and tracks north of the project site. The Santa Barbara Railroad Depot is located north of the site. An existing residential structure is located immediately adjacent to the southwest abutment and a second residence is located within about 30 feet of the northwest corner of the bridge. A tourist hostel is located adjacent to the southeast corner of the bridge. Mission Creek generally flows through the site from north to south toward the Pacific Ocean located about ½-mile from the existing bridge.

1.2 PROPOSED IMPROVEMENTS

On the basis of discussions with the project team, we understand the existing sandstone abutment walls and concrete lining in the creek will remain in place and the abutments for the new bridge will be constructed adjacent to but behind the existing abutments. Therefore the new bridge is anticipated to have a span similar to, but slightly longer than the existing 60-foot-span. We anticipate the overall length of the bridge also will be similar to the existing bridge; however, the project team has discussed an option for not reconstructing a triangular portion of the bridge north of Yanonali Street.

In addition to the bridge replacement project at Chapala Street, we understand that the Corps of Engineers, together with the County and City of Santa Barbara, are designing flood control improvements on Mission Creek. As part of the overall Lower Mission Creek Flood Control Project, we understand that a box culvert will be constructed east of and immediately adjacent to the west abutment of the new bridge. The culvert will connect to an existing culvert beneath the UPRR corridor on the north and daylight at Mission Creek immediately east of the bridge. The existing culvert is currently bulkheaded at the upstream and downstream ends. At this time, the distance that will separate the proposed replacement bridge and the new box culvert is not known.

2. PREVIOUS DATA

As-built data for the bridge is not available. However, we anticipate the bridge abutments are founded on shallow foundations. Selected published and unpublished documents and information that we reviewed or used to assist in our evaluation are referenced in this report.

3. WORK PERFORMED

3.1 PURPOSE

The purpose of this report is to provide preliminary geotechnical engineering input to the preliminary design of the Chapala Street Bridge Seismic Retrofit/Replacement Project.

3.2 SCOPE

Our scope of work for the preliminary foundation report consisted of the following tasks:

- Site visits to observe the general site conditions,
- Generalized soil and groundwater conditions based on existing data;
- Preliminary seismic data for use with Caltrans design methods including causative fault, peak bedrock acceleration, depth to bedrock, soil profile type, and a site response spectra developed in accordance with the Caltrans Seismic Design Criteria;
- Preliminary seismic information and qualitative assessment of geologic hazards such as seismicity, fault ground rupture hazards, liquefaction potential, and seismic settlement;
- Suitable foundation types for support of the new structure; and
- Preliminary opinions regarding construction considerations related to excavation characteristics of the soils encountered, adjacent structures, and CIDH pile construction.

3.3 GENERAL CONDITIONS

Fugro prepared the conclusions, recommendations, and professional opinions of this report in accordance with the generally accepted geotechnical principles and practices at this time and location. This warranty is in lieu of all other warranties, either expressed or implied. This report was prepared for the exclusive use of Drake Haglan & Associates and their authorized agents only. It may not contain sufficient information for the purposes of other parties or other uses. If any changes are made in the project as described in this report, the conclusions and recommendations contained in this report should not be considered valid unless Fugro reviews the changes and modifies and approves, in writing, the conclusions and recommendations of this report. This report and the drawings contained in this report are intended for design-input purposes; they are not intended to act as construction drawings or specifications.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements, or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

Soil and rock deposits can vary in type, strength, and other geotechnical properties between points of observations and exploration. Additionally, groundwater and soil moisture conditions also can vary seasonally or for other reasons. Therefore, we do not and cannot have a complete knowledge of the subsurface conditions underlying the site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed by construction.

4. SITE CONDITIONS

4.1 REGIONAL GEOLOGY

The project is situated in the Transverse Ranges geomorphic province of southern California. The Transverse Ranges province is oriented generally east-west, which is oblique to the general north-northwest trending structural trend of California mountain ranges. The Transverse Ranges province extends from the Los Angeles Basin westward to Point Arguello, and is composed of Cenozoic- to Mesozoic-age sedimentary, volcanic, igneous, and metamorphic rocks. The Santa Ynez Mountains and adjacent lowlands are comprised of sedimentary rocks and soil materials ranging in age from Cretaceous to recent.

Structural geology in the Santa Barbara and Goleta area consists of mountain and foothill areas underlain by generally south-dipping bedrock units and low lying coastal plain areas generally underlain by younger and older alluvium. The area generally includes a series of subparallel, east-west trending faults and folds that are the result of north-south compressional tectonics. The faults and folds roughly parallel the Santa Ynez Mountains and are present inland and offshore in the Santa Barbara Channel.

The general geology in the project area consists of a low-lying coastal plain of Quaternary-age alluvium unconformably overlying a thick sequence of Tertiary-age sedimentary rocks. Local geologic conditions in the project area as mapped by Dibblee (1986) are shown on Plate 3 – Regional Geologic Map.

4.2 SUBSURFACE CONDITIONS

4.2.1 General

Our preliminary understanding and interpretation of the subsurface conditions in the project area are based on cone penetration test sounding (CPT) data acquired for this project. Planned soil borings for the project and geotechnical laboratory testing for the project had not been completed at the time this report was written. We also reviewed and evaluated data acquired by Fugro for a portion of the Lower Mission Creek Flood Control Project between the UPRR corridor and the north side of Highway 101 (Fugro (2009). Explorations for that project consisted of advancing cone penetration test (CPT) soundings and excavating, logging and sampling hollow-stem auger drill holes. Locations of the explorations performed for this study and selected exploration locations from Fugro (2009) that are proximal to the site are shown on Plate 4 –Subsurface Exploration Plan. Our interpretation of the subsurface conditions is primarily based on CPT data. The logs of the CPT soundings performed for this project are presented in Appendix A – Subsurface Exploration. Data for the selected CPT soundings and drill holes performed for Fugro (2009) are provided in Appendix B – Fugro (2009) Subsurface Exploration Data.

General subsurface conditions at the site are anticipated to consist of artificial fill overlying an interbedded sequence of alluvial deposits (Qal). Descriptions of the geologic units and soil conditions are presented below.

4.2.2 Artificial Fill

From our review of the CPT soundings acquired for this project, observations made during construction of the box culvert in the UPRR corridor, and data from Fugro (2009), we interpret the soil materials in the upper 10 feet and behind the abutment walls to be artificial fill consisting of medium stiff lean clay containing layers of medium dense silty sand.

4.2.3 Alluvium (Qal)

On the basis of the available data, we interpret that alluvial soils are present below the surficial artificial fill and extend to the maximum depth explored of about 100 feet below the ground surface. The alluvium generally consists of medium stiff (to locally soft) fine-grained soils (lean clay and sandy lean clay) interbedded with loose to medium dense coarse-grained soils (clayey sand and silty sand) to a depth of about 40 feet below the ground surface. Below that depth the soils appear to consist of stratified dense to very dense sand and stiff to very stiff lean clay with the thickness of the various strata ranging from a few feet thick to about 25 feet thick.

The undrained shear strength of the fine-grained soils above a depth of about 40 feet is anticipated to range from 500 to 1,000 psf. The undrained strength of the lower stiff to very stiff fine-grained soils is anticipated to range from about 1,500 to 3,000 psf.

4.3 GROUNDWATER CONDITIONS

The depth to groundwater was also measured in holes formed by the CPT soundings after the rods were withdrawn. Groundwater was measured in the CPT sounding holes at depths of about 7 to 9 feet below the ground surface. The depth to groundwater measured in Fugro (2009) drill holes DH-1 and DH-3 (located in the park/RR depot parking lot area) ranged from about 9 to 9-1/2 feet below the ground surface in November 2008. On the basis of the available data, we recommend the depth to groundwater at the site for preliminary planning and design purposes be assumed to be about 7 feet below existing street grade.

We note that groundwater levels and zones of perched water in the lower and upper channel areas can vary over time in response to environmental changes and land use changes. As such, groundwater levels at the time of construction or in the future could differ from the values obtained in this study.

4.4 IDEALIZED SOIL PROFILE

Soil conditions at the proposed abutment locations are anticipated to consist of interbedded alluvium consisting of medium stiff to stiff lean clay underlain by medium dense to very dense silty sand with layers and strata of stiff to very stiff lean clay. The groundwater level is anticipated to be at about 7 to 9 feet below the ground surface.

Assumed idealized soil profiles at the proposed abutment locations are outlined in Table 1 – Idealized Subsurface Conditions. We used the following idealized soil profile for the preliminary analyses performed for this report.

Table 1. Idealized Subsurface Conditions

Approximate Elevation (feet) (assumed ground surface el. of 18 ft)	Material	Total Unit Weight (pcf)	Estimated Undrained Shear Strength (psf)	Estimated Friction Angle (degrees)
+18 to +9	Dense silty Sand	125		30
+9 to –12	Medium Stiff Lean Clay	125	750	0
-12 to -22	Medium Dense Silty Sand	125	0	30
-22 to -25	Stiff Lean Clay	125	1500	0
-25 to -42	Dense to Very Dense Silty Sand	125	35	0
-42 to -57	Stiff to Very Stiff Lean Clay	125	2000	0
-57 to -82	Dense to Very Dense Silty Sand	125	35	0

5. SEISMIC SETTING

The project site is in a seismically active region of southern California. We performed a search of controlling faults in the area in accordance with current Caltrans Seismic Design Criteria and utilizing Caltrans ARS Online (Caltrans 2009a) and the 2007 Caltrans Deterministic PGA Map. Caltrans ARS Online is a web-based tool operated through the Caltrans website and is based on the Caltrans 2007 Fault Database that is continuously updated. ARS online displays information for faults included in the Caltrans 2007 Fault Database and calculates both deterministic and probabilistic acceleration response spectra (ARS) for any location in California as described in Appendix B of the Caltrans Seismic Design Criteria (Caltrans 2009b). ARS Online was first used to identify potential controlling faults in the site vicinity. Table 2 – Potential Controlling Faults presents a list of potential controlling faults closest to the site identified using ARS Online and site coordinates corresponding to Latitude 34.4128 and Longitude 119.69270.

We also used ARS Online to estimate strong ground motion and develop a design ARS for the site as discussed in subsequent sections of this report.

Table 2. Potential Controlling Faults

Fault Name	Fault Type[1]	R_x Distance (mi)[2]	Maximum Magnitude (MMax)[3]
Mesa Rincon Creek Fault	Reverse	0.4	6.8
San Jose Fault	Reverse	0.4	6.3
North Channel Slope Fault	Blind Thrust	0.8	7.4
Mission Ridge Arroyo Parida Fault	Reverse	2.5	7.2
More Ranch Fault	Reverse	2.5	7.2
Red Mountain Fault	Reverse	3.6	7.0

1: Fault type per Caltrans 2007 Fault Database. 2: Horizontal distance to the fault trace (fictitious fault trace for sites offset from the fault) or surface projection of the top of rupture plane measured perpendicular to the fault from the site per ARS Online and Caltrans Geotechnical Services Design Manual Version 1.0. 3: MMax values per ARS Online and Caltrans 2007 Database.

Brief descriptions of potentially controlling faults identified by ARS Online closest to the site are provided below.

Mesa Rincon Creek Fault. The Mesa Rincon Creek Fault identified on ARS Online is mapped northeast of the project site and dips to the south at 45 degrees. The site is located on the hanging wall of the fault directly over the fault plane.

San Jose Fault. The San Jose Fault identified on ARS Online is mapped northwest of the project site and dips to the south at 45 degrees. The site is located on the hanging wall of the fault and is offset from the fault.

North Channel Slope Fault. The North Channel Slope Fault identified on ARS Online is mapped south of the project site and dips to the northeast at 26 degrees. The fault is a blind thrust fault with the top of rupture plane located 6.2 miles below the ground surface. The site is located on the hanging wall side of the fault.

Mission Ridge Arroyo Parida Fault. The Mission Ridge Arroyo Parida Fault identified on ARS Online is mapped north of the project site and dips to the south at 70 degrees. The site is located on the hanging wall side of the fault.

5.1 STRONG GROUND SHAKING

In accordance with the Caltrans Seismic Design Criteria, we used ARS Online to estimate strong ground motion and develop a design acceleration response spectra (ARS) for the project site. As discussed previously, ARS Online calculates both deterministic and probabilistic ARS for any location in California based on the Caltrans Seismic Design Criteria for faults included in the Caltrans 2007 Fault Database. Caltrans seismic design procedures include a comparison of the ARS Online estimated probabilistic ARS with the 2008 USGS Interactive

Deaggregation Tool (Beta) (USGS, 2008) when the estimated shear wave velocity V_{s30} for the site is less than or equal to 300 meters/second. The development of design ARS for the site is discussed in Section 5.3 of this report.

Based on results of Caltrans seismic design procedures using ARS Online and comparison with results generated by the 2008 USGS Interactive Deaggregation Tool (Beta), a maximum considered (975-year return period) peak ground acceleration of 0.64g is estimated for the site. The Mesa Rincon Creek Fault is the controlling fault for the deterministic ARS.

5.2 GROUND SURFACE RUPTURE

The site is not located in an Alquist-Priolo Earthquake Fault Zone as defined by the State of California. The closest significant faults to the project site identified in ARS Online are the Mesa Rincon Creek, San Jose, and North Channel Slope Faults located approximately 0.4, 0.4, and 0.8, miles from the site, respectively. On the basis of that information, in our opinion, the potential for ground surface rupture from faulting is considered to be low.

5.3 SEISMIC DESIGN CRITERIA

5.3.1 Design Response Spectra

A design acceleration response spectrum (ARS) curve for the site was developed using ARS Online and the requirements set forth in Appendix B of the Caltrans Seismic Design Criteria. The Caltrans Seismic Design Criteria also requires use of the 2008 USGS Interactive Deaggregation calculator (Beta version) as a tool during the development of the design probabilistic ARS curve when the estimated shear wave velocity V_{s30} for the site is less than or equal to 300 meters/second. We used CPT data from a previous study performed for this project just north of the site near the railroad station (Fugro, 2009). We estimated shear wave velocities for materials encountered in the CPT soundings by using correlations to CPT tip resistance and undrained shear strength and shear wave velocity presented in the Caltrans Geotechnical Services Design Manual (Caltrans, 2009c). An average shear wave velocity of 670 feet/sec was estimated for the top 100 feet of soil at the site. According to Appendix B of the Caltrans Seismic Design Criteria, a site with a shear wave velocity V_{s100} (V_{s30} in metric units) of 670 feet/sec (205 meters/second in metric units) corresponds to a Soil Profile Type D.

The deterministic and probabilistic spectra resulting from the ARS Online analysis for 5 percent damping are shown on Plate 4 – Acceleration Response Spectra – 5% Damping. The design deterministic ARS curve was controlled by the Mesa Rincon Creek Fault. In accordance with Caltrans guidelines, we compared the site-specific deterministic ARS curve to the minimum deterministic ARS curve for California (defined by Caltrans as magnitude 6.5 vertical strike-slip event occurring at 7.5 miles from the site). The site-specific deterministic ARS curve is higher for all periods than the Caltrans minimum deterministic ARS curve for California. In accordance with Caltrans guidelines, the design ARS curve is taken as the upper envelope of the deterministic and probabilistic ARS curves. The design ARS curve is controlled by the probabilistic spectrum shown on Plate 4, and has an estimated peak ground acceleration of 0.64g.

5.4 LIQUEFACTION HAZARD

Liquefaction is the loss of strength that can occur in saturated coarse-grained soils during earthquake seismic shaking. The susceptibility of a granular soil to liquefaction is a function of the gradation, relative density, and fines content of the soil. Susceptibility to liquefaction generally decreases with increasing mean grain size, relative density, fines content and clay-size fraction of the fines, and the age of the deposit.

Liquefaction is a phenomena principally associated with granular soils. However, some studies show that liquefaction and/or strength loss can occur in some fine-grained soils during a seismic event (Moss et al. 2006). Based on the soil classification, shear strength, and Atterberg limit data acquired for this project and the conclusions regarding liquefaction provided in geotechnical reports prepared by others for nearby projects, we have assumed that the fine-grained silty clay to clay soils encountered in our explorations would not experience significant liquefaction-related strength loss. If more conclusive findings regarding the liquefaction potential of the fine-grained materials at the site are necessary, site-specific dynamic testing should be performed. Dynamic testing of the on-site soil materials is beyond our scope of work for the project.

There are a number of potential consequences that occur as a result of liquefaction. When the shaking continues after the onset of liquefaction, liquefaction can produce a number of ground effects (e.g., sand boils, settlement, lurching, and lateral displacement). Liquefaction also can cause a loss of bearing capacity of shallow foundations, loss of lateral support and additional vertical loads for deep foundations, and lateral ground spreading. In general, the longer the duration of strong shaking after the initiation of liquefaction, the greater the consequences.

5.4.1 Method of Evaluation

Our evaluation of liquefaction potential at the project site was performed using data from the CPT soundings, drill holes, and results of laboratory testing. Liquefaction potential was evaluated using NCEER (Youd and Idriss, 2001) guidelines for a magnitude 6.9 event with a peak horizontal ground acceleration of 0.64g. This event corresponds to an earthquake with an estimated 5% probability of exceedance in 50 years or with a return period of 975 years.

5.4.2 Liquefaction Potential

The soil materials encountered in the CPT soundings performed for the project generally consisted of about 30 feet of medium stiff lean clay soils underlain by medium dense to very dense sand and silty sand with interbedded with layers and strata of stiff to very stiff lean clay.

Data from liquefaction analyses using the CPT data indicates there is a potential for liquefaction to occur in strata of medium dense silty sand at the site between depths of about 32 and 37 feet below the existing ground surface (approx El. -14 to -19 feet). A similar layer of medium dense silty sand was encountered in the CPT sounding performed by Bengal

Engineering (Bengal Engineering 2010). The results also suggest that liquefaction could occur in a few, generally isolated sandy layers below that depth. However, analyses using the CPT data indicate that granular materials encountered below a depth of about 40 feet generally have a low potential for liquefaction.

Our analyses also indicate that liquefaction of the some of the medium stiff fine-grained soil layers encountered above 40 feet could also occur. However, for the reasons described above, in our opinion these fine-grained soils have a low potential to liquefy under the design seismic event.

In our opinion, the liquefaction hazard and the associated consequences of liquefaction at the project site are anticipated to be similar throughout the project region (that is tens of feet from the channel). Therefore, in our opinion, the potential for liquefaction at the site could be considered to be a regional hazard. Consequences of liquefaction on the proposed bridge foundation should be considered in the project design. Our evaluation of the potential consequences of liquefaction on the project is provided below.

5.4.3 Potential Consequences of Liquefaction

Liquefaction Settlement. On the basis of our evaluations, we estimate that ground surface settlements of about 2 inches could occur from liquefaction under the earthquake scenarios considered for the project. The settlements are anticipated to generally result from liquefaction of the soils in the upper 40 feet. Lew and Martin (1999) suggest that differential settlements from liquefaction at sites underlain by relatively uniform conditions can be estimated as about one half the estimated total settlement. Because the site conditions appear to be relatively uniform, in our opinion, preliminary estimates of differential settlements from liquefaction can be assumed equal to one half of the estimated total settlement or about 1 inch. We note that our estimated settlement is on the basis of our interpretations of CPT logs.

Sand Boils. Sand boils are formed when granular material in a liquefied soil layer (generally near the ground surface) is forced to the ground surface by the buildup of soil pore water pressures. The formation of sand boils can result in general ground surface subsidence due to the ejection of soil material from the subsurface. The data suggests that potentially liquefiable soils are present within about 30 feet of the ground surface (about 20 feet below the flow line of the creek). On that basis, the potential hazard associated with sand boils occurring at the site as a result of liquefaction is probably low to moderate.

Lateral Spreading. Lateral spreading (decoupling and sliding of soil layers at the interface of a liquefied soil layer) results in lateral deformation and cracking of the ground surface. The presence of the Mission Creek channel creates a "free face condition," and results in a higher potential for lateral spreading when compared to level or gently sloping, uniform ground conditions. Because the potentially liquefiable soils are located at depths of between 32 and 37 feet (about 20 to 25 feet below the flow line of Mission Creek) and are overlain by medium stiff clayey soils, in our opinion, the potential for lateral spreading at the site is considered low to moderate and may not need to be considered in the design of the project.

Downdrag. As discussed, liquefaction of the soil layer between about 32 and 37 feet below the ground surface could result in total settlements of about 2 inches. Settlements of that magnitude have the potential to induce downdrag loads on deep foundation elements. Analyses performed to evaluate the axial capacity of potential foundation systems (driven piles and cast-in-drill hole piles) were performed for this preliminary study and the results are discussed in more detail below. On the basis of those analyses, non-factored skin friction downdrag loads for the foundation types evaluated (14-inch precast concrete pile, 16-inch OD steel pipe pile, a and 30-inch-diameter CIDH pile) are anticipated to range from about 35 tons for the 14-inch driven concrete pile, 25 tons for the 16-inch-diameter pipe pile, and about 50 tons for the 30-inch-diameter CIDH pile.

6. PRELIMINARY FOUNDATION CONSIDERATIONS

Due to the proximity of existing residential structures that will remain along Mission Creek, the design of the bridge will likely involve design and construction methods that minimize impacts to that and other nearby structures. We anticipate the bridge abutments will be founded on cast-in-drill-hole (CIDH) or driven piles and that site grades in the project area will not be modified significantly by the reconstruction project.

6.1 FOUNDATION TYPES

6.1.1 Shallow Foundations

It is our opinion that shallow foundations are not suited for this project due o the anticipated high foundation loads and the presence of soft to medium stiff clay soils above a depth of about 30 feet. Deep foundations will allow for the bridge loads to be transferred to the more suitable firmer alluvial soils at depth.

6.1.2 Driven Piles

Vertical Axial Capacity. Driven pile foundations consisting of precast concrete or steel piles are considered potentially feasible for support for the structure. Potential negative aspects of using driven piles for foundation support consist of noise impacts to the neighborhood and vibration impacts to the residential structures and youth hostel located within about 30 to 50 feet of the existing abutments. Noise and vibration impacts from pile driving could potential be reduced by using open-ended pipe piles and using a combination of driving and center drilling to drive (or push) the pile to the required tip elevation. In any event, detailed pre-construction condition surveys of the existing structures should be performed together with vibration monitoring during pipe installation.

For preliminary purposes, we evaluated the axial capacity of two selected pile types consisting of a 14-inch precast concrete pile and a 16-inch O.D, 15-inch I.D. steel pipe pile. The analyses were performed using the computer program APILE 5.0 (Ensoft 2009). On the basis of our evaluation, we anticipate that 14-inch-square precast concrete piles approximately 60 feet in length (approximate tip elevation of -57 feet assuming a pile head elevation of +3 feet) would likely provide static nominal axial capacities of 200 tons. For a 16-inch-diameter steel pipe pile, a pile length of about 67 feet is anticipated to provide a static, nominal axial capacity of 200 tons.

To achieve a nominal axial capacity of 200 tons considering the potential downdrag from liquefaction (estimated to be about 35 to 25 tons), about 5 feet of additional pile length would likely be required for the concrete pile. About 10 feet of additional pile length would likely be required for the steel pipe pile. The estimated axial capacity of the 30-inch-diameter drilled shaft was evaluated using idealized soil profile for the site together with the computer program APILE v5.0 (Ensoft 2009a).

Using a minimum resistance factor of 0.7, as indicated in Caltran's amendments to Table 10.5.5.2.3-1 of the AASHTO Bridge Design Specifications (2008), would yield an estimated stress limit state capacity of 140 tons for a driven precast concrete or steel pipe pile (200 tons nominal resistance x resistance factor of 0.7).

Lateral Capacity. Lateral capacity of driven piles will be evaluated during final design.

6.1.3 Drilled Piles

Vertical Axial Capacity. In our opinion, drilled piles consisting of Caltrans standard cast-in-drilled-hole (CIDH) piles are also a potentially feasible alternative for supporting the new bridge abutments. The presence of relatively shallow groundwater and soil conditions consisting of stratified and layered fine- and coarse-grained alluvial soils could lead to potential caving problems during drilling. Temporary casings and or drilling fluid will likely be required during CIDH pile construction to help support sandy alluvial soils and minimize caving. Variable drilling conditions are expected and localized layers or seams of gravel and cobbles may be encountered.

We expect that 30-inch-diameter CIDH pile approximately 50 to 55 feet in length (approximate tip elevation of -47 to -52 feet assuming a pile head elevation of +3 feet) would likely provide a static nominal axial capacity of about 200 tons. To achieve a nominal axial capacity of 200 tons considering the potential downdrag from liquefaction (estimated to be about 50 tons), about 10 to 15 feet of additional pile length would likely be required. The estimated axial capacity of the 30-inch-diameter drilled shaft was evaluated using idealized soil profile for the site together with the computer program SHAFT v6.0 (Ensoft 2009b).

A minimum resistance factor of 0.7 for drilled shafts is specified in Caltran's amendments to Table 10.5.5.2.4-1 of the AASHTO Bridge Design Specifications (2008). However, because difficult drilling conditions may be encountered during drilling and that casing and or drilling fluid will likely be required for construction of drilled shafts, we recommend the resistance factor be limited to a value of 0.5. Using a resistance factor of 0.5, the stress limit resistance for a drilled shaft with a nominal resistance of 200 tons would be 100 tons.

Lateral Capacity. Lateral capacity of driven piles will be evaluated during final design.

6.2 CONSTRUCTION CONSIDERATIONS

6.2.1 Temporary Slopes and Shoring

Temporary slopes should be braced or sloped according to the requirements of OSHA. In accordance with OSHA requirements, the contractor should be responsible for job site safety, for reviewing the soil conditions encountered during construction, and for the design of temporary slopes and shoring systems. Within the expected depth of excavation, the subsurface conditions are likely to consist of gravelly alluvium and older alluvium materials. Based on OSHA guidelines and the alluvial materials encountered, temporary slopes should be excavated to 1½ h:1v or be shored to support Type C soil conditions.

6.2.2 Groundwater and Dewatering

Groundwater conditions are discussed in Section 4.3 of this report. Groundwater was encountered approximately 7 to 9 feet below the existing ground surface in our CPT sounding holes and was measured in our previous Fugro (2009) drill holes excavated north of the UPRR corridor. Construction of CIDH piles using wet placement methods and temporary casing will likely be needed during construction of the CIDH piles.

7. CLOSURE

This Preliminary Foundation Report was prepared for Drake Haglan & Associates that their authorized agents for use in planning and preliminary design of the proposed Chapala Street Bridge Retrofit/Replacement Project.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, or atmosphere, although samples for water quality testing were obtained and submitted for analysis. Any statements, or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessments.

In performing our professional services, we have used generally accepted geologic and geotechnical engineering principles and have applied that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers currently practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report.

8. REFERENCES

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Fugro (2009), "Geotechnical Engineering Report, Lower Mission Creek Improvements, Santa Barbara, California," DRAFT report prepared for HDR Engineering, Fugro Project No. 3161.018, dated January 16

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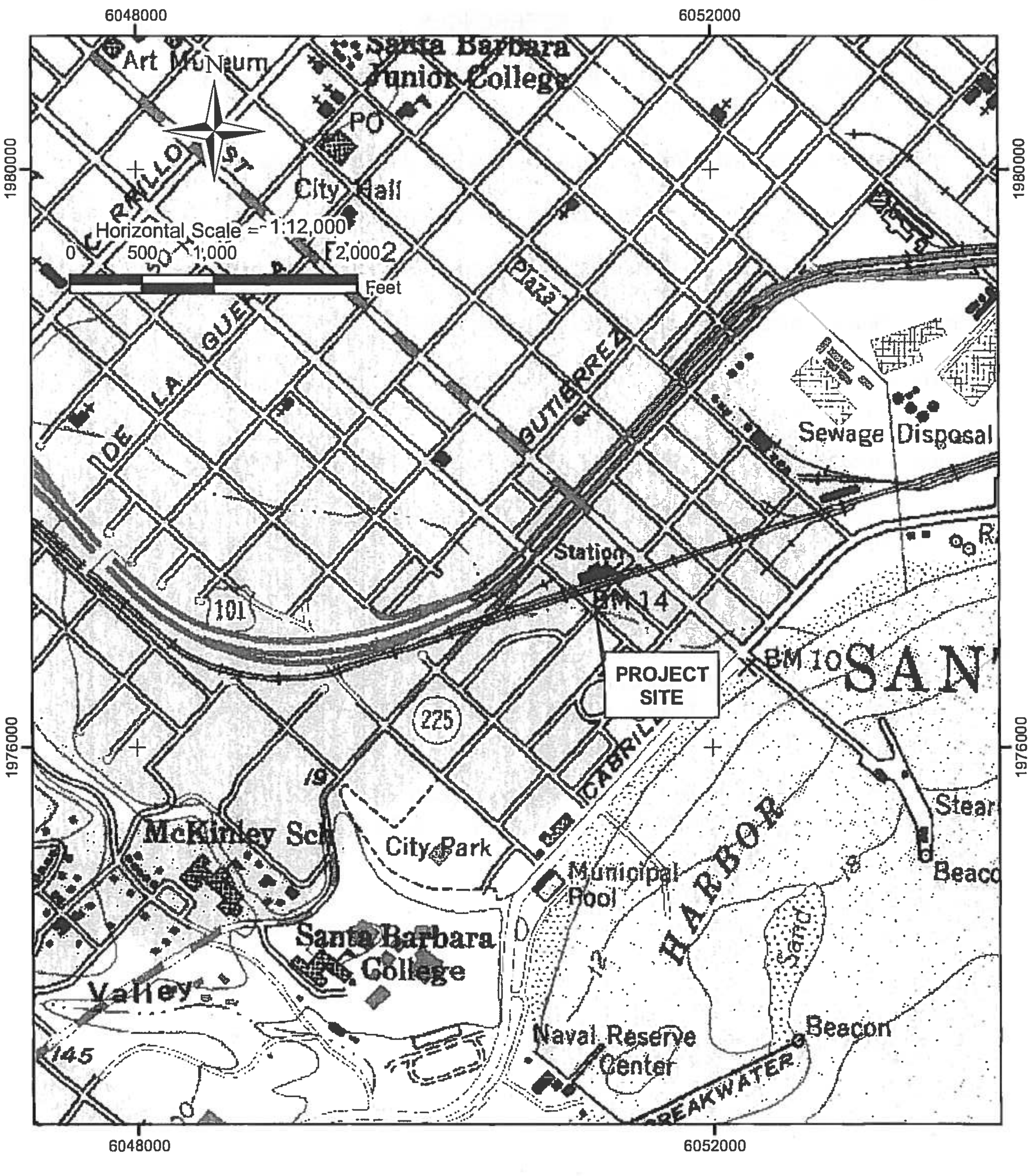
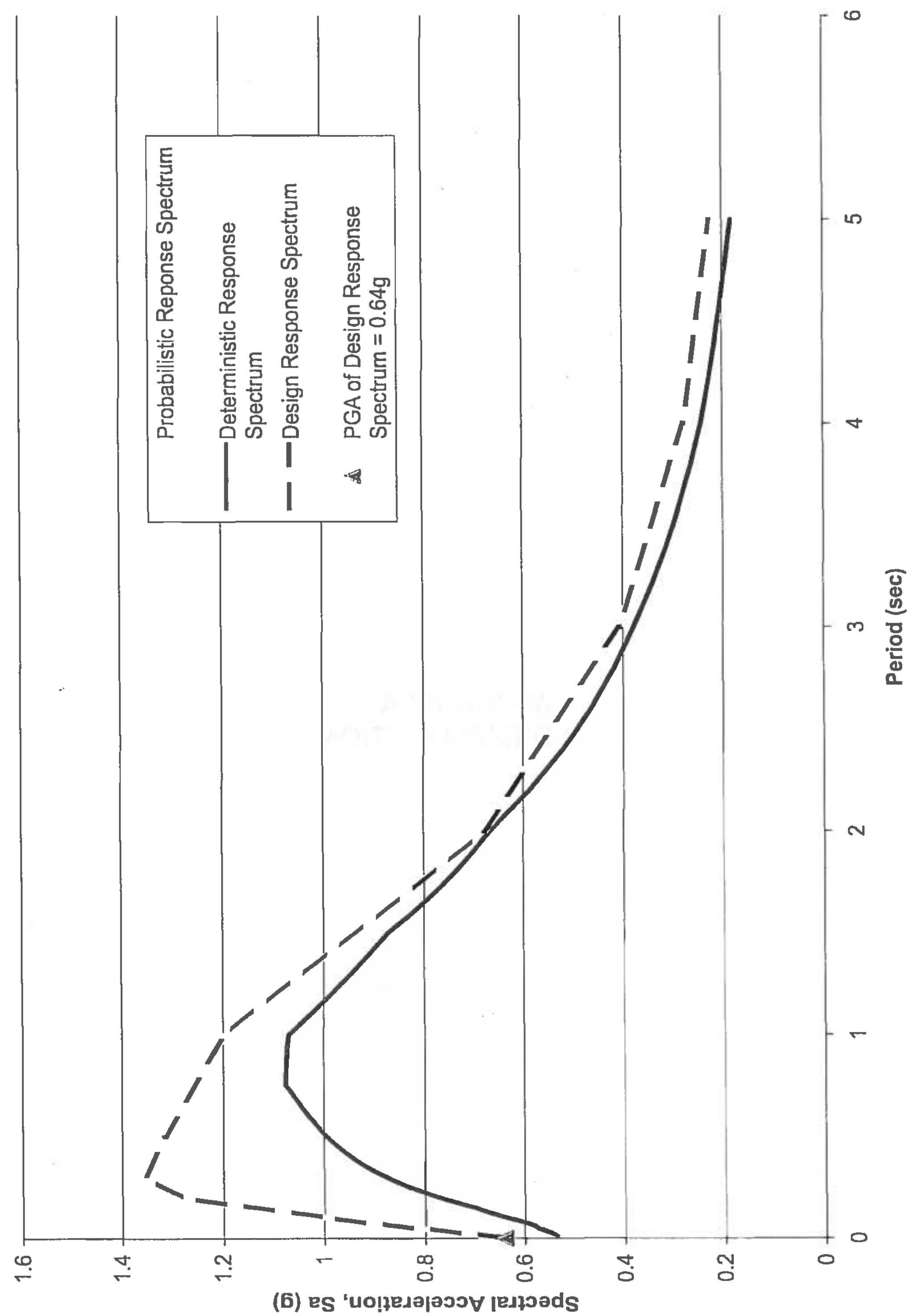


SITE MAP

Chapala Street Bridge
Retrofit/Replacement Project
Santa Barbara, California

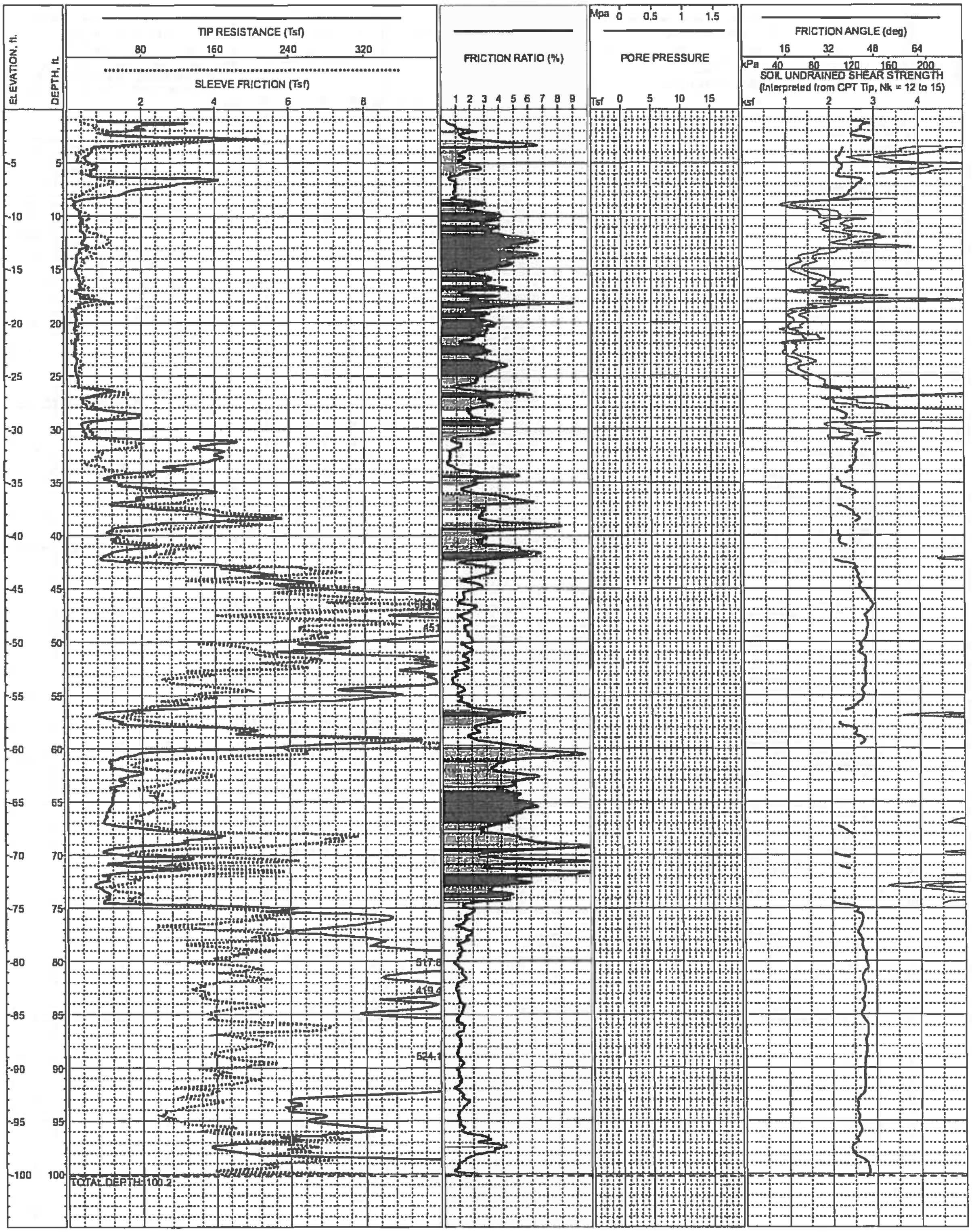


ACCELERATION RESPONSE SPECTRA - 5% Damping
Chapala Street Bridge Retrofit/Replacement Project
Santa Barbara, California

PLATE 4

APPENDIX A
FIELD EXPLORATION

Drake Haglan & Associates
Project No. 3713.001



LOCATION: E 6,051,131, N1,976,927
SURFACE EL: 0.0ft +/- (MSL)
COMPLETION DEPTH: 100.2ft
TESTDATE: 3/5/2010

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: G S Denlinger

LOG OF CPT NO: CPT-1

Chapala Street Bridge Retrofit/Replacement Project
Santa Barbara, California

PLATE A

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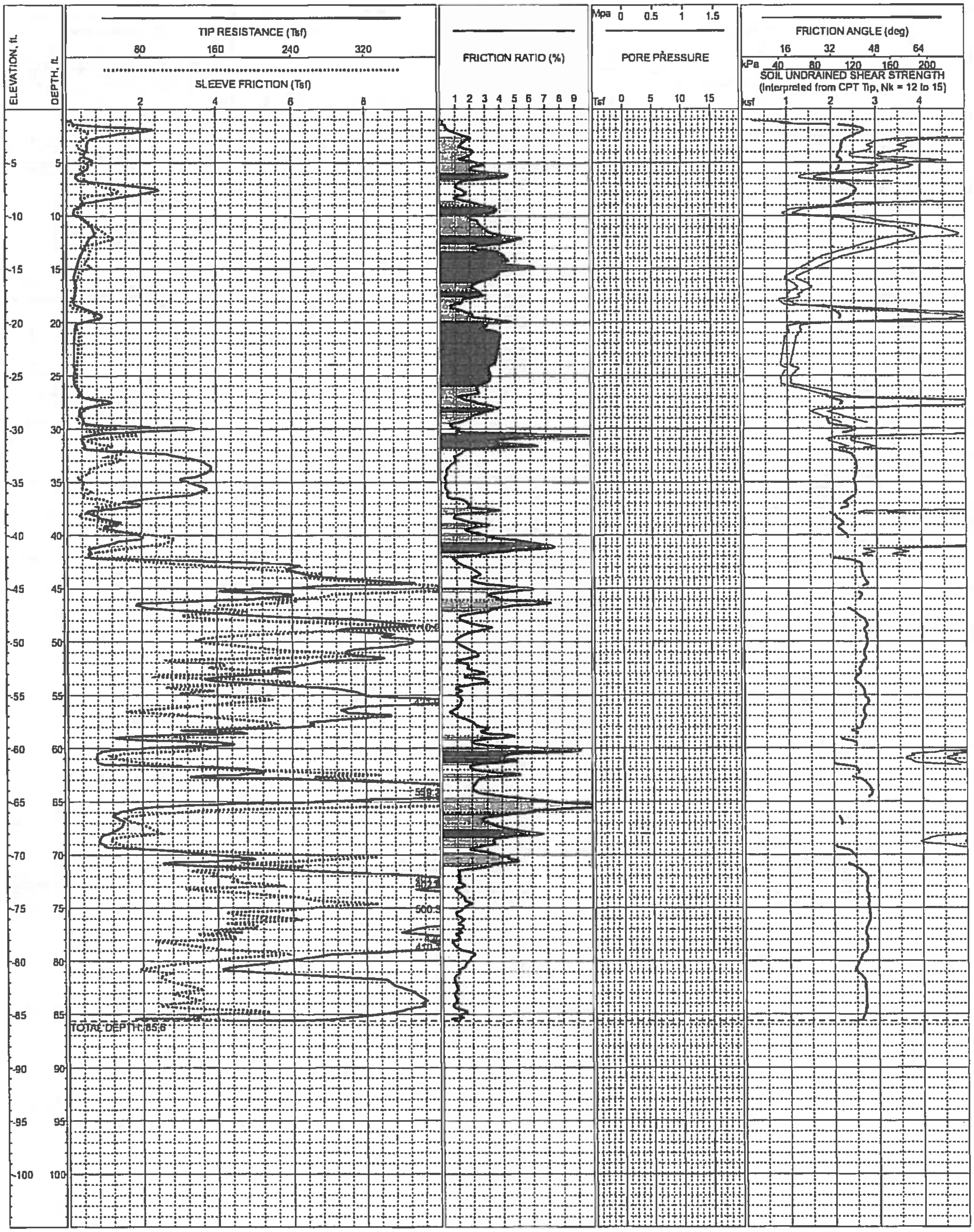

LOCATION: E 6,051,207, N1,976,905
SURFACE EL: 0.0ft +/- (MSL)
COMPLETION DEPTH: 85.6ft
TESTDATE: 3/5/2010

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: G S Denlinger

LOG OF CPT NO: CPT-2

Chapala Street Bridge Retrofit/Replacement Project
Santa Barbara, California

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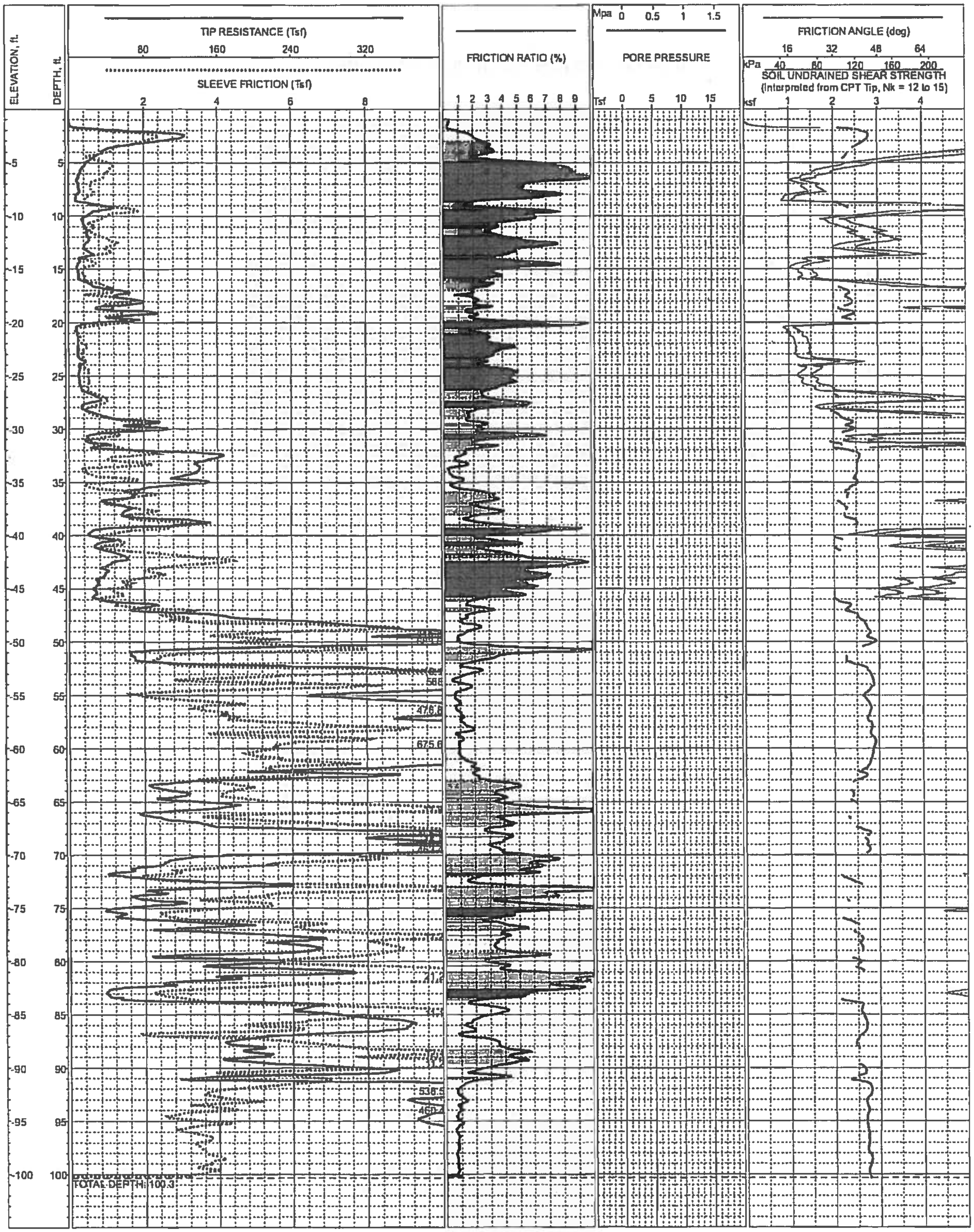
Drake Haglan & Associates
Project No. 3713.001



LOCATION: E 6,051,136, N1,976,974
SURFACE EL: 0.0ft +/- (MSL)
COMPLETION DEPTH: 100.3ft
TESTDATE: 3/5/2010

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Geosciences
REVIEWED BY: G S Denlinger

LOG OF CPT NO: CPT-3

Chapala Street Bridge Retrofit/Replacement Project
Santa Barbara, California

PLATE A

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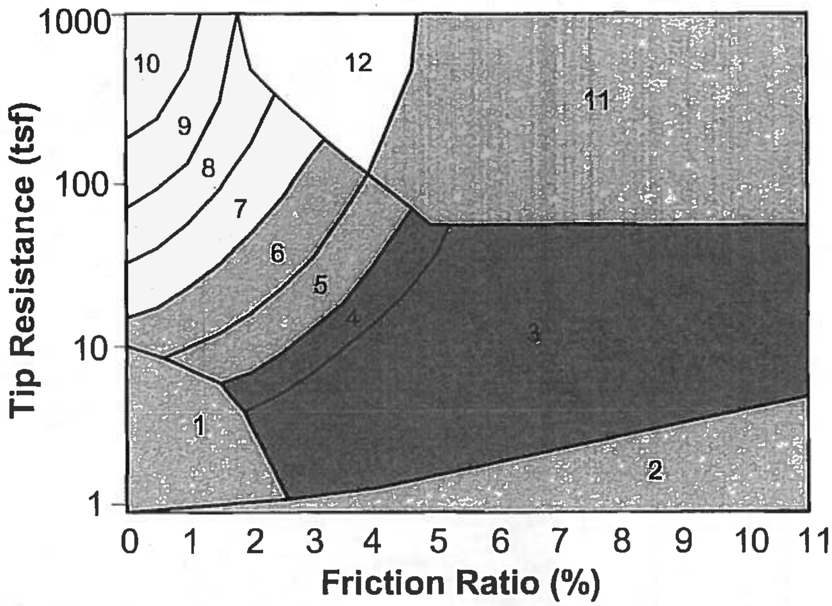
COLOR LEGEND FOR FRICTION RATIO TRACES



Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

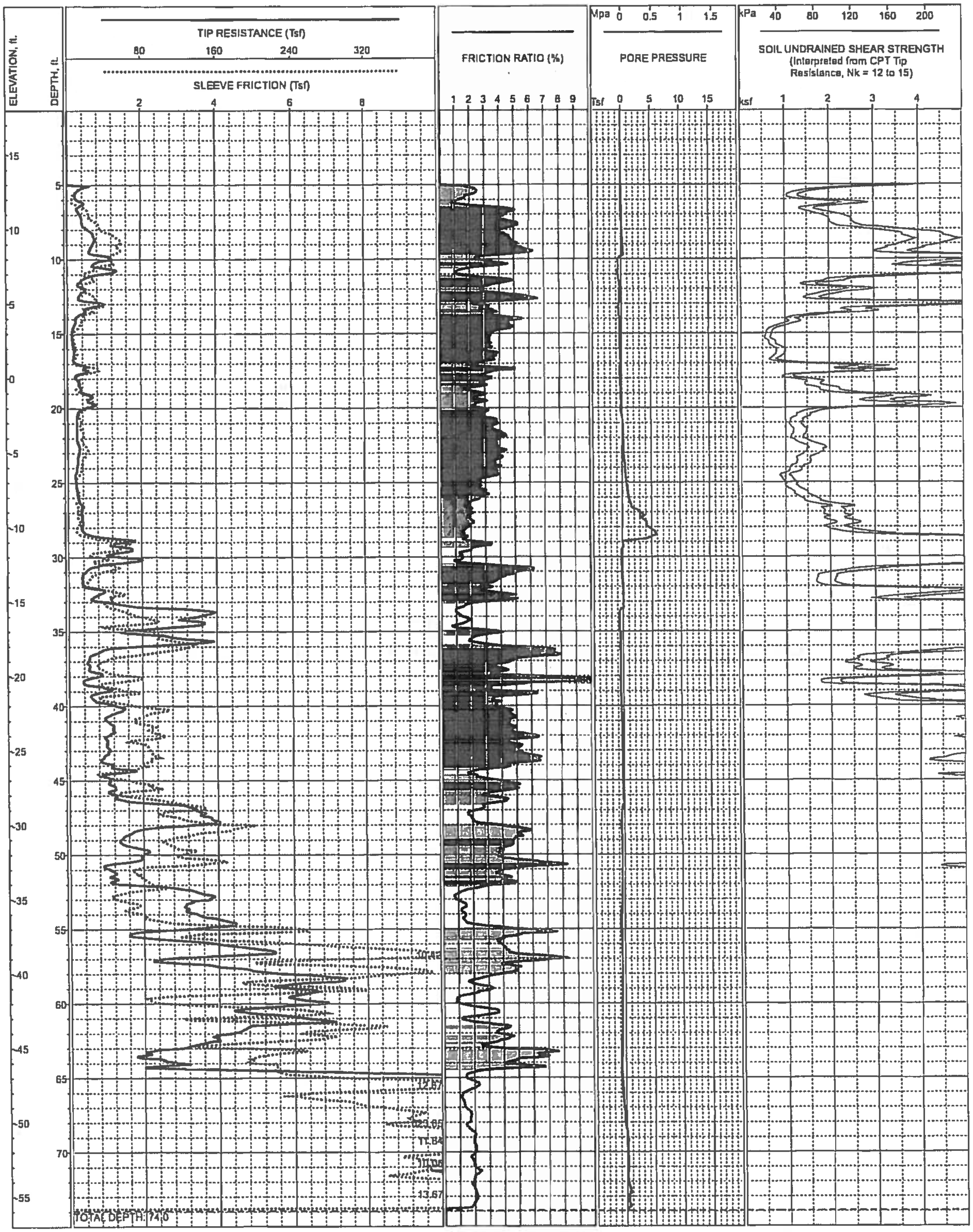
*overconsolidated or cemented

CPT CORRELATION CHART
(Robertson and Campanella, 1984)

KEY TO CPT LOGS
Lower Mission Creek Improvements
Santa Barbara County, California

APPENDIX B
FUGRO (2009) SUBSURFACE EXPLORATION DATA

HDR Engineering
Project No. 3161.018



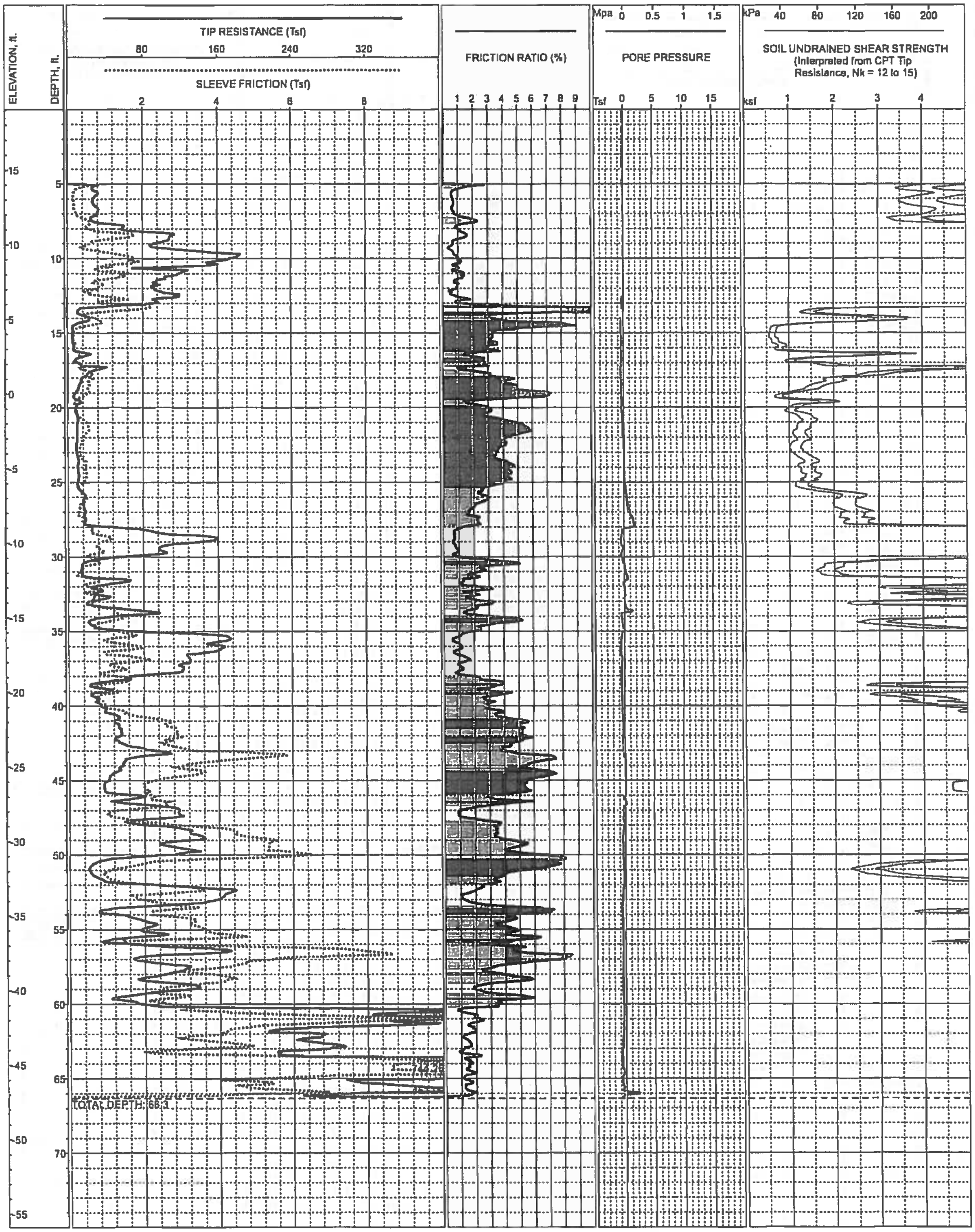
LOCATION: Railroad depot parking lot, west of Chapala St., near DH-3
SURFACE EL: 18.0ft +/- (MSL)
COMPLETION DEPTH: 74.0ft
DEPTH TO WATER: 12.5 ft
TESTDATE: 10/24/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: G S Denlinger

LOG OF CPT NO: CPT-1

Lower Mission Creek Improvements
Santa Barbara, California

FUGRO



LOCATION: Railroad depot parking lot, approximately 75 feet souheast of Montecito St.
SURFACE EL: 19.1ft +/- (MSL)
COMPLETION DEPTH: 66.3ft
DEPTH TO WATER: 12 ft
TESTDATE: 10/24/2008

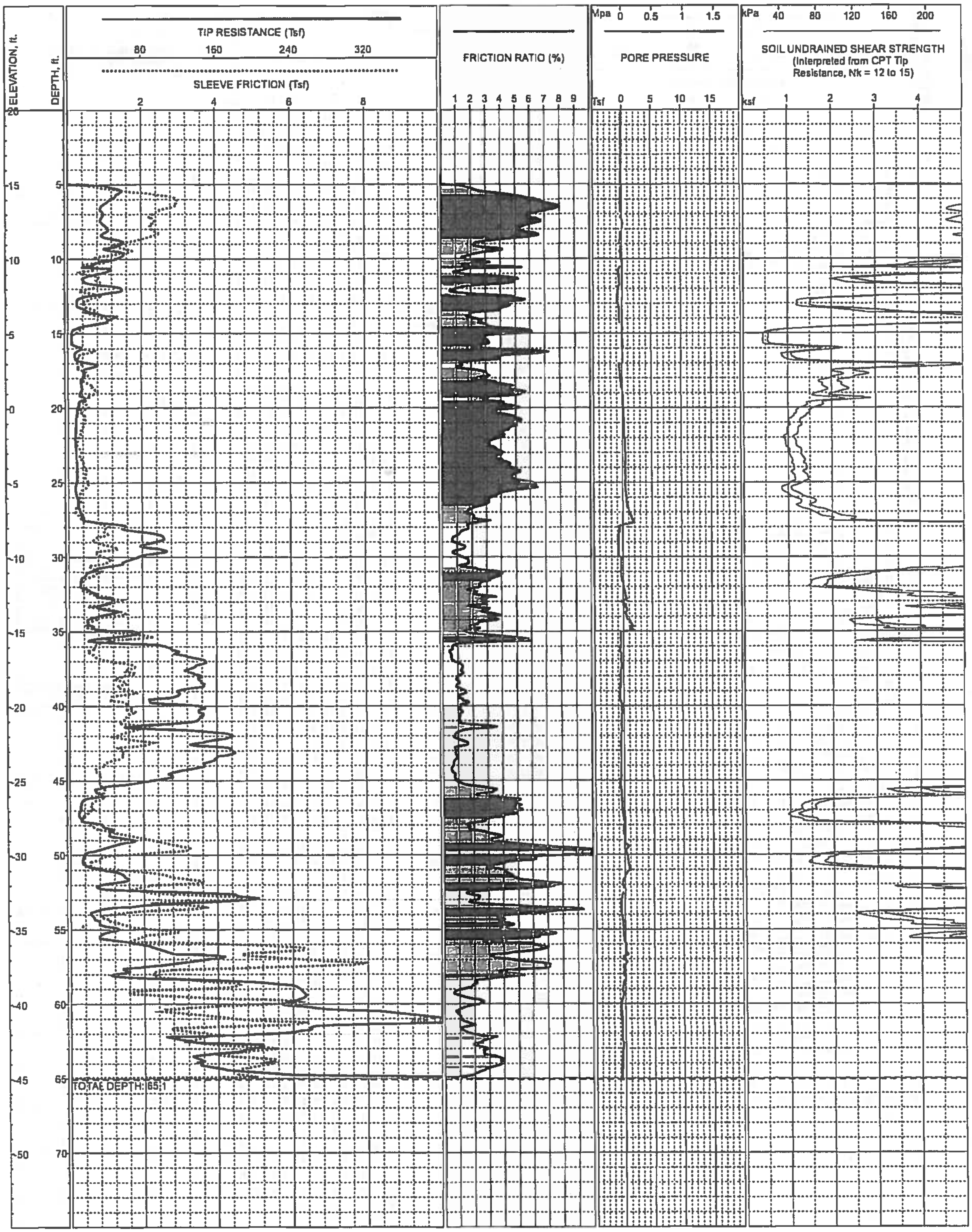
EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: G S Denlinger

LOG OF CPT NO: CPT-2

Lower Mission Creek Improvements
Santa Barbara, California

N:\Projects\3161_HDREngineering\3161-018_LowerMissionCreekImp\Explorations\CPT\2008\Logs\Logs_LowMissCreekImp_Su_2008_10_28\MXD\Logs_LowMissCreekImp_Su_2008_10_28.mxd,01/15/2009,kshell

HDR Engineering
Project No. 3161.018



LOCATION: Southeast side of Montecito St., near DH-1
SURFACE EL: 20.1ft +/- (MSL)
COMPLETION DEPTH: 65.1ft
DEPTH TO WATER: 12.5 ft
TESTDATE: 10/24/2008

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Fugro Consultants, Inc.
REVIEWED BY: G S Denlinger

LOG OF CPT NO: CPT-3

Lower Mission Creek Improvements
Santa Barbara, California

N:\Projects\3161_HDREngineering\3161-018_LowerMissionCreekImp\Explorations\CPT\2008\Logs\Logs_LowMissCreekImp_Su_2008_10_28\MXD\Logs_LowMissCreekImp_Su_2008_10_28.mxd,01/15/2009,ksheil

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: SE side of Montecito Street, See Plate 2 - Subsurface Exploration Location Plan SURFACE EL: 20 ft +/- (rel. MSL datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ARTIFICIAL FILL (af) Silty SAND (SM): dark yellow brown, slightly moist, fine to medium sand							
~18	2					- with gravel at ~2-1/2'							
~16	4												
			1		(32)	**ALLUVIUM (Qal)** Sandy Lean CLAY (CL): very stiff, dusky brown, moist, fine sand, trace coarse, angular sand, trace pin-sized voids and rootlets near ~5', some moderate brown mottling	134	118	14				p 4.5+
~14	6		2		13				14				
~12	8					- turns to moderate brown at ~7', with trace silty sand lenses, mottled pale brown, trace rootlets and fine, subrounded gravels at ~8-1/2'							
~10	10				(9)	- wet at ~9-1/2' medium stiff, w/ sandy silt lenses and silty sand lenses							
			3			Lean CLAY (CL): medium stiff, moderate brown w/ olive gray mottles, wet, w/ rootlets	128	105	22				
~8	12		4		8				31		39	17	
~6	14					- turns to olive gray, mottled moderate brown, near ~13', w/ some sand							
			5		(5)	- lens of sandy lean clay at ~15', olive gray w/ moderate brown mottles, fine sand			25				
~4	16												
~2	18												
~0	20		6		(9)	- lens of fat clay at ~19-1/2', olive gray to dark gray, high plasticity, w/ rootlets near ~19'	124	96	29				p 0.5 t 0.9
~-2	22												
~-4	24												
			7		6								
~-6	26		8		(13)	- sandy at ~26'	129	105	23				p 0.5
~-8	28					Silty SAND (SM): medium dense, dark gray, wet, fine sand							
~-10	30		9A 9B		14	SILT with sand (ML): stiff, dark gray, wet, fine sand, trace rootlets, trace clay pockets							
~-12	32												
~-14	34												
			10		15	- lens of clayey sand at ~35'			20				
~-16	36					Silty SAND (SM): medium dense, dark gray, wet, fine to medium sand							
~-18	38												
					27								

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 44.0 ft
DEPTH TO WATER: GW not measured, sample wet at ~9-1/2'
BACKFILLED WITH: Cuttings to ~25', Install observation well at ~25'
DRILLING DATE: November 6, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger, hand auger to 5 ft
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: K Nelson
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-1

Lower Mission Creek Improvements
Santa Barbara, California

PLATE A-2a

HDR Engineering
Project No. 3161.018

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: SE side of Montecito Street, See Plate 2 - Subsurface Exploration Location Plan SURFACE EL: 20 ft +/- (rel. MSL datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
			11						23	31			
~22	42												
~24	44		12			- heaving sands in sampler at ~44', terminate hole at 44-1/2 due to flowing sands							
~26	46												
~28	48												
~30	50												
~32	52												
~34	54												
~36	56												
~38	58												
~40	60												
~42	62												
~44	64												
~46	66												
~48	68												
~50	70												
~52	72												
~54	74												
~56	76												
~58	78												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 44.0 ft
DEPTH TO WATER: GW not measured, sample wet at ~9-1/2'
BACKFILLED WITH: Cuttings to ~25', install observation well at ~25'
DRILLING DATE: November 6, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger, hand auger to 5 ft
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: K Nelson
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-1

Lower Mission Creek Improvements
Santa Barbara, California

PLATE A-2b

LOCATION: train station parking lot, See Plate 2 - Subsurface Exploration Location Plan

SURFACE EL: 18 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
16	2					**ARTIFICIAL FILL (af)** Silty SAND (SM): dark yellow brown, slightly moist, fine sand, with fine gravel - lean clay layer at ~2', moderate borwn, with fine gravel - trace rootlets at ~3'							
14	4												
12	6		1A 1B		(6)	- fat clay lens at ~5-1/4', soft, with rootlets, lens of dark yellow orange sand at ~5-1/2'	99 123	83 97	19 27				t 0.4 p 2.0 t 1.2
			2		11	**ALLUVIUM (Qal)** Sandy Lean CLAY to Lean CLAY with sand (CL): medium stiff, dusky brown, slightly moist, with trace rootlets			16		32	16	
10	8												
					(12)	- stiff at ~7', less sand, moderate brown to dark yellow brown mottled olive gray and light brown, slightly moist							
8	10		3			- wet at ~9'	128	108	19	23			
6	12				8	- silty sand layer at ~10', fine to medium sand, trace coarse sand							
4	14		4A 4B		(7)	- dark yellow brown mottled yellow brown and olive gray, silty lens near ~15'	125 126	100 103	25 23		28	12	t 0.4 t 0.4 u 0.5
2	16												
0	18												
-2	20		5A 5B		(5)	- lens of silty fine sand at ~19', very loose to soft, , very fine sand, turns to lean clay near ~20-1/2', with rootlets	 125	 99	25 25				
-4	22												
-6	24		6		6	- lens of fat clay at ~24', medium stiff, dark gray, high plasticity, trace rootlets							p 0.5
-8	26		7A 7B		(11)	- sandy clay at ~27', fine to medium sand, trace coarse sand							
-10	28												
-12	30		8A 8B		12	SILT (ML): stiff, dark gray, wet, lens of silty fine sand at ~29-1/2', becomes clayey near 30-1/2'							p 0.8
-14	32												
-16	34												
-18	36												
-20	38												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 30.5 ft
DEPTH TO WATER: GW not measured, sample wet at ~9'
BACKFILLED WITH: Cuttings to ~25', install observation well at ~25'
DRILLING DATE: November 7, 2008

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger, hand auger to 5 ft
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: K Nelson
CHECKED BY: G S Denlinger

LOG OF BORING NO. DH-3

Lower Mission Creek Improvements
Santa Barbara, California

PLATE A-4

LOCATION: The drill hole location referencing local landmarks or coordinates

SURFACE EL: Using local, MSL, MLLW or other datum

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLES	BLOW COUNT / REC"/DRIVE"	MATERIAL DESCRIPTION	
-12	2		1		25	Well graded GRAVEL (GW)	COARSE GRAINED
-14	4		2		(25)	Poorly graded GRAVEL (GP)	COARSE GRAINED
-16	6					Well graded SAND (SW)	COARSE GRAINED
-18	8		3		(25)	Poorly graded SAND (SP)	COARSE GRAINED
-20	10		4		(25)	Silty SAND (SM)	COARSE GRAINED
-22	12		5		18"/ 30"	Clayey SAND (SC)	COARSE GRAINED
-24	14		6			Silty, Clayey SAND (SC-SM)	COARSE GRAINED
-26	16					Elastic SILT (MH)	FINE GRAINED
-28	18		7			SILT (ML)	FINE GRAINED
-30	20		8		20"/ 24"	Silty CLAY (CL-ML)	FINE GRAINED
-32	22		9		(25)	Fat CLAY (CH)	FINE GRAINED
-34	24		10		30"/ 30"	Lean CLAY (CL)	FINE GRAINED
-36	26					CONGLOMERATE	ROCK
-38	28		11		20"/ 24"	SANDSTONE	ROCK
-40	30		12			SILTSTONE	ROCK
-42	32		13			MUDSTONE	ROCK
-44	34					CLAYSTONE	ROCK
-46	36					BASALT	ROCK
-48	38					ANDESITE BRECCIA	ROCK
						Paving and/or Base Materials	

General Notes

Soil Texture Symbol

Sloped line in symbol column indicates transitional boundary

Samplers and sampler dimensions
(unless otherwise noted in report text) are as follows:

Symbol for:

1	SPT Sampler, driven 1-3/8" ID, 2" OD
2	CA Liner Sampler, driven 2-3/8" ID, 3" OD
3	CA Liner Sampler, disturbed 2-3/8" ID, 3" OD
4	Thin-walled Tube, pushed 2-7/8" ID, 3" OD
5	Bulk Bag Sample (from cuttings)
6	CA Liner Sampler, Bagged
7	Hand Auger Sample
8	CME Core Sample
9	Pitcher Sample
10	Lexan Sample
11	Vibracore Sample
12	No Sample Recovered
13	Sonic Soil Core Sample

Sampler Driving Resistance

Number of blows with 140 lb. hammer, falling 30" to drive sampler 1 ft. after sealing sampler 6"; for example,

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of sealing
86/11"	After driving sampler the initial 6" of seating, 36 blows drove sampler through the second 6" interval, and 50 blows drove the sampler 5" into the third interval
50/6"	50 blows drove sampler 6" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

Classification of Soils per ASTM D2487 or D2488

Geologic Formation noted in bold font at the top of interpreted interval

Strength Legend

Q = Unconfined Compression
u = Unconsolidated Undrained Triaxial
t = Torvane
p = Pocket Penetrometer
m = Miniature Vane

Water Level Symbols

- Initial or perched water level
- Final ground water level
- Seepages encountered

Rock Quality Designation (RQD) is the sum of recovered core pieces greater than 4 inches divided by the length of the cored interval.

KEY TO TERMS & SYMBOLS USED ON LOGS